CALCULATION/OTHER DOCUMENTS COVER SHEET

CALCULATION NUMBER CALC – 771 - BS - 000189 **Rev.** 0

Section 1: IDENTIFICATION

1. WCF or /Authorization Project Number	2. Project Title	3. Page
CAEAA102	**B771 TUNNELS STRUCTURAL ANALYSIS FOR THE PREDICTION OF LONG TERM CONDITION**	1 of 21

3. System Identification (See SX-164, Plant System and Component Identification and Labeling) NA	4. Other (Type of document, e.g., Studies, Conceptual Design Report, Design Criteria, etc.) Capacity Analysis
6. Natural Phenomena Hazard Performance Category (PC) Number ☒ PC-0 / NA ☐ PC-1 ☐ PC-2 ☐ PC-3	7. Building Number B771

8. Engineering Discipline(s) Involved with Calculation:

STRUCTURAL

Section 2: SIGNATURES FOR A CALCULATION

	Discipline	Print Name	Sign	Date
9. Designer(s)	*Structural*	Keith MacLeod	Keith MacLeod	09/25/03
10. Checker(s)	Structural	Tom Frank	Thomas H Frank	09/25/03
11. Independent Verifier (for PC-0/NA and PC-1)	Structural	Tom Frank	Thomas H Frank	09/25/03
12. Peer Reviewer (for PC-2 and PC-3)	NA			
13. Responsible Engineering Manager	PCE	Tim Humiston	T Humiston	9/29/03
14. Classification Review	DC	U/NU	W. J. McANDREW	9/25/03

Section 3: SIGNATURES FOR OTHER DOCUMENTS

	Discipline	Print Name	Sign	Date
15. Preparer				

Section 4: REVISION SUMMARY

16. Description	17. Affected Pages

DOES NOT CONTAIN
OFFICIAL USE ONLY INFORMATION
Name/Org. J A Nesheim EMCBC Classification Date 10/6/05

ADMIN RECORD

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CALCULATION CONTROL NUMBER: CALC – 771 – BS – 000189 - (REV. 0)

1. **IWCP/Authorization Project Number:** <u>**CAEAA102**</u>

2. **Calculation Title:** <u>**B771 TUNNELS STRUCTURAL ANALYSIS FOR THE PREDICTION OF LONG TERM CONDITION**</u>

3. **Calculation Description:**

   The site is considering leaving the concrete portions of tunnels from B771 in place and not removing them for the final site closure. There two tunnels that are from B771 to B776, and from B771 to the stack. This calculation addresses two factors that will be involved with this consideration, which are as follows:

   1. What is the projected number of years that the tunnel will remain standing before it begins to collapse.
   2. What will be the depression in the ground surface when the tunnel does collapse.

   Therefore, an analysis of the tunnel structure's present strength and condition is needed to determine what the future long term condition of the tunnel may be. Based on the results of the /analysis a projection can be made as to how many years before the tunnel begins to collapse. The analysis is based on the tunnel loaded only with the soil overburden that is presently on the tunnel. The tunnel will not be subject to any vehicle traffic. The analysis is also based on the groundwater rising to above the tunnels at least part of every year. The tunnels will therefore be exposed to the corrosive effects of water.

4. **Natural Phenomena Hazard Performance Category:** **NA** - it can be reasonably assumed that if an earthquake does occur it will not effect the tunnel, because the tunnel is buried and supported all around by soil.

5. **Calculation Objectives (List):**

   The objective is to calculate the strength of the tunnel without steel rebar reinforcement and just with the strength of the concrete. This will give an indication of whether the tunnel can support its own weight and overburden over a long period of time once the reinforcement has completely corroded. The natural groundwater flows are expected to rise above the tunnels at least part of each year. This will expose the tunnel to water and over a long enough period of time the reinforcement will corrode.

   The objective also is to approximate the effects on the ground surface after the tunnel has collapsed.

6. **List Methods used for Calculation:** Standard engineering design practice and by engineering methods of the (ACI) American Concrete Institute.

7. **List Assumptions used:** The assumption that the ground water will rise above the tunnels for at least part of the year, is base on the report Summary of Integrated Modeling For the Area Surround Building 771, By Bob Prucha (9-23-2003).

8. Identify References:

8.1 Drawings (attached):

Concrete Connection Tunnel B. G. 76 to 71	- 17205-001A - Bldg. (RF-76-172205)
Tunnel Details Bldg. 71	- 00S15-001A – Bldg. (RF-71-S15-0)
Topographic Map Zone B-5	- 15510-1015 - Site Civil

8.2 ACI 318-89 American Concrete Institute 1989 Edition.

8.3 Summary of Integrated Modeling For the Area Surround Building 771
By Bob Prucha (9-23-2003)

9. Identify Applicable Design Related AB Documents: **NA**

10. Body of Calculation: Refer to the following calculation pages.

11. Calculation Conclusion:

B771 Tunnels Structural Analysis for the Prediction of Long Term Condition

Present Strength & Condition of Tunnels

The tunnels are in good condition with no cracks or evidence of corrosion. But, after site closure the tunnels are expected to be expose, inside and out, to ground water for at least part of each year. The ground water will seep into the concrete and corrode the reinforcement until, it some period of time the reinforcement becomes ineffective. The final strength of the tunnel is then dependent on the uncracked ultimate tensile (rupture) strength of the concrete. When the concrete eventually deteriorates and cracks the roof loses all strength and will collapse. The collapse of the tunnel roof will cause the soil overburden to fill the tunnels and there will be a depression on the ground surface.

The calculations conclude that all of the tunnels, except the North end of the tunnel from 771 to 776, can support its own weight and the soil overburden without reinforcement. The soil overburden is about 24 feet at the North end of the tunnel between 771 & 776. The calculations show that the concrete strength of the tunnel roof can support almost that amount of soil overburden. Therefore, when that part of the tunnel does collapse, the soil above the tunnel will have a arching effect and will support it's self. There will be no visible effect on the ground surface. The only visible effect on the surface will be when the tunnel finally collapse where the overburden is shallow enough that it will not arch. That will be all of the tunnel to the stack and the South end of the tunnel to 776.

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11. Calculation Conclusion (continued):

Long Term Durability of Building B771 Tunnels

The long term durability of the tunnels is first dependent on the period of time that will take to corrode the reinforcement so that it becomes ineffective. Because of the good condition of the tunnels and the probability that the tunnels will only be expose to ground water part of each year, it may take approximately 500 years or longer for the reinforcement to corrode. Once the reinforcement has corroded the concrete could support the soil overburden for at least another 500 years or longer, even the North portion of the tunnel between 771 & 776 because of the arching effect of the soil.

Because, the concrete of the tunnel has the strength to support its own weight and the soil overburden, the tunnel could remain intact for many years. Therefore, a conservative engineering estimate would be that the tunnels could continue exist without failing for at least **1,000 to 2,000 years.**

Depression in Soil After the Eventual Collapse of the Tunnels

When the tunnel does eventually collapse the soil overburden will settle into the tunnels. There is enough soil overburden above the tunnel between 771 & 776, so that the soil will settle into and fill the tunnel and leave a depression in the grade at the surface. The tunnel from 771 to the stack does not have enough soil overburden to completely fill the tunnel. Therefore, there will be a open trench after this tunnel collapses. Although, after 1,000 to 2,000 years the surrounding soils will probably drain and fill in the tunnel.

Tunnel between 771 & 776

The settlement of the soil will spread at approximately 45 degree angle from the top corner of the tunnel roof. The soil will settle into the tunnel at the South end, and the soil will arch and bridge the tunnel at the North end. There will be a depression in the surface at the South end and then decrease to no depression probably some where in the middle of the tunnel where the soil begins to arch and bridge over the tunnel. Therefore, the surface depression will be approximately as follows:

South End - **4.5 ft. deep x 22 ft. wide** Middle to the North End – **no depression**

Tunnel from 771 to Stack

The soil overburden and roof concrete will fill the tunnel to approximately a 4 ft. depth. Which means there could be a open trench at the tunnel 6.0 ft. deep x 8.0 ft. wide. Eventually the surrounding soil will fill the trench, but it could take several hundreds of years.

If there are any questions please give me a call.

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B771 TUNNELS STRUCTURAL ANALYSIS

SOIL OVERBURDEN CALC.
(REFER TO REFERENCE DRAWINGS.)

TUNNEL FROM B771 TO B776

NORTH (B771) END

T.O. FLR. ELEV. = 5950.0'
TUNNEL HEIGHT* = 11.25'
T.O. TUNNEL = 5961.25'

* HEIGHT = DIST. FROM FLR. TO T.O. TUNNEL

T.O. GRADE = 5985.00
− 5961.25

SOIL OVERBURDEN = 23.75'

SOUTH (B776) END

T.O. FLR. EL. = 5967.00
TUNNEL HEIGHT* = 11.00
T.O. TUNNEL = 5978.00'
T.O. GRADE = 5984.00'
− 5978.00

SOIL OVERBURDEN = 6.00'

B771 TUNNELS STRUCTURAL ANALYSIS

SOIL OVERBURDEN CALC.

(REFER TO REFERENCE DRAWINGS.)

TUNNEL FROM B771 TO STACK

WEST (B771) END

* HEIGHT = DIST. FROM FLR. TO T.O. TUNNEL

T.O. FLR. ELEV. = 5,974.5'
TUNNEL HEIGHT* = 10.83'
T.O. TUNNEL = 5,985.33'
T.O. GRADE = 5,987.00'
− 5,985.33'

SOIL OVERBURDEN = 1.67'

EAST (STACK) END AT LOW POINT

T.O. FLR. EL = 5967.3'
TUNNEL HT = 10.83'
T.O. TUNNEL = 5978.13'
T.O. GRADE = 5982.00
SOIL OVERBURDEN = 3.87'

EAST (STACK) END AT STACK.

T.O. FLR. EL. = 5966.0'
TUNNEL HT. = 14.0'
T.O. TUNNEL = 5980.0'
T.O. GRADE = 5982.0"
SOIL OVERBURDEN = 2.0'

B771 TUNNELS STRUCTURAL ANALYSIS

LOAD ON TUNNELS ROOF

DENSITY OF SOIL (CONSERVATIVE)

DRY WEIGHT = 100 pcf USE SOIL γ = 110 pcf

WET WEIGHT = 120 pcf

TUNNEL FROM 771 TO 776

NORTH END - SOIL WT. = 110 pcf × 23.75' = 2,612.5 psf
CONC. WT. = 150 pcf × 1.3' = 195.0 psf
2,807.5 psf

SOUTH END
SOIL WT. = 110 pcf × 6.0' = 660 psf
CONC. WT. = 150 pcf × 1.0 = 150 psf
810 psf

TUNNEL FROM 771 TO STACK

WEST END - SOIL WT = 110 pcf × 1.67' = 183.7 psf
CONC. WT. = 150 × .83' = 124.5 psf
308.2 psf

EAST END @ LOW POINT
SOIL WT. = 110 × 3.87' = 425.7
CONC. WT. = 150 × .83' = 124.5 psf
550.2 psf

EAST END @ STACK
SOIL = 110 × 2.0' = 220 psf
CONC. = 150 × .83' = 124.5 psf
344.5 psf

B771 TUNNELS STRUCTURAL ANALYSIS

OVERBUREN ROOF MOMENT

TUNNEL FROM 771 TO 776

SPAN = 8.0' ASSUME ROOF BETWEEN SIMPLE & FIXED AT SUPPORTS.

NORTH END

$$M_{MAX} = \frac{w\ell^2}{10} = \frac{2{,}807.5\,psf \times (8.0')^2}{10} = \underline{17{,}968}\ ^{lb\text{-}FT.}$$

SOUTH END

$$M_{MAX} = \frac{810\,psf\,(8.0')^2}{10} = \underline{5{,}184}\ ^{lb\text{-}FT}$$

TUNNEL FROM 771 TO STACK — SPAN = 8.0'

WEST END $M_{MAX} = \frac{308.2\,psf\,(8.0')^2}{10} = \underline{1{,}972.5}\ ^{lb\text{-}FT}$

EAST END LOW POINT

$$M_{MAX} = \frac{550.2\,psf\,(8.0')^2}{10} = \underline{3{,}521.3}\ ^{lb\text{-}FT}$$

EAST END AT STACK SPAN = 6.0'

$$M_{MAX} = \frac{344.2\,(6.0)^2}{10} = \underline{1{,}239.1}\ ^{lb\text{-}FT}$$

B771 TUNNELS STRUCTURAL ANALYSIS

CONCRETE STRENGTHS

CONCRETE STRENGTH OF BOTH TUNNELS

$f'_c = 3000$ psi MIN. AT 28 DAYS

NOTE: CONCRETE CONTINUES TO GAIN STRENGTH AFTER 28 DAYS & $f'_c = 3000$ psi IS MINIMUM, ACTUAL STRENGTH ARE BETWEEN 4000 psi & 3000 psi.

THEREFORE: ASSUME $f'_c = 3,500$ psi

TENSILE STRENGTH OF CONCRETE (MODULUS OF RUPTURE)

REF: ACI 9.5.2.3 (9-9)

$f_r = 7.5\sqrt{f'_c} = 7.5\sqrt{3,500} = 443.7$ psi

B771 TUNNELS STRUCTURAL ANALYSIS

TUNNEL CONCRETE ROOF CAPACITY
WITHOUT REINFORCEMENT (ASSUMED CORRODED)

TUNNEL FROM 771 TO 776

NORTH END   ROOF THICKNESS $t = 15''$

SECTION MODULUS PER 12" WIDTH

$$S = \frac{12''(15'')^2}{6} = 450.0 \text{ IN}^3$$

CRACK MOMENT CAPACITY $M_{CR} = f_r \times S$

$$M_{CR} = 443.7\text{psi} \times 450.0\text{in}^3 = 199{,}665 \text{ lb-in}$$

$$= \underline{16{,}638.7 \text{ lb-ft}} < M_{MAX} = 17{,}968 \text{ lb-ft}$$

CONCRETE CAN NOT SUPPORT OVERBURDEN WITHOUT REINFORCEMENT.

SOUTH END   ROOF THICKNESS $t = 12''$

$$S = \frac{12(12)^2}{6} = 288.0 \text{ in}^3$$

$$M_{CR} = 443.7\text{psi} \times 288.0\text{in}^3 \times \frac{\text{FT}}{12\text{IN}}$$

$$= \underline{10{,}648.8 \text{ lb-ft}} > M_{MAX} = 5{,}184 \text{ lb-ft}$$

CONCRETE CAN SUPPORT OVERBURDEN WITHOUT REINFORCEMENT.

B771 TUNNELS STRUCTURAL ANALYSIS

TUNNEL CONCRETE ROOF CAPACITY
WITHOUT REINFORCEMENT (ASSUMED CORRODED)

TUNNEL FROM 771 TO STACK

WEST END — ROOF THICKNESS $t = 10''$

$$S = \frac{12''(10'')^2}{6} = 200\ \text{IN}^3$$

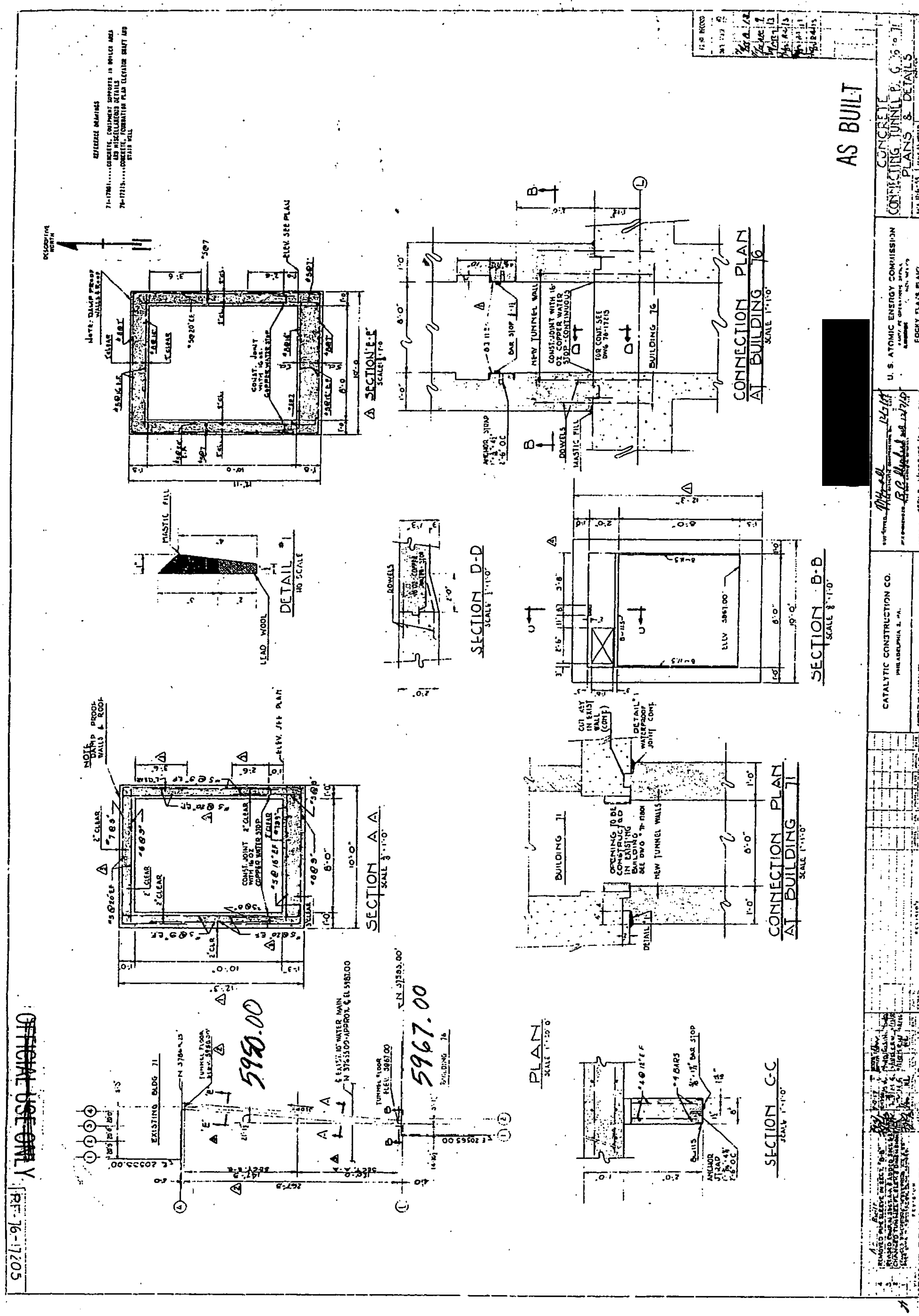
$$M_{CR} = 443.7\,\text{psi} \times 200\,\text{IN}^3 \times \frac{\text{FT}}{12\,\text{IN}} = 7{,}395^{\text{lb-F}} > M_{MAX} = 1{,}972.5^{\text{lb-F}}$$

EAST END — $t = 10''$ $S = 200\,\text{IN}^3$

$$M_{CR} = 7{,}395^{\text{lb-F}} > M_{MAX} = 3{,}521.3^{\text{lb-FT}}$$

CONC. CAN SUPPORT OVERBURDEN WITHOUT REINFORCEMENT

CALC-771-BS-000189 (PAGE 12 of 21)



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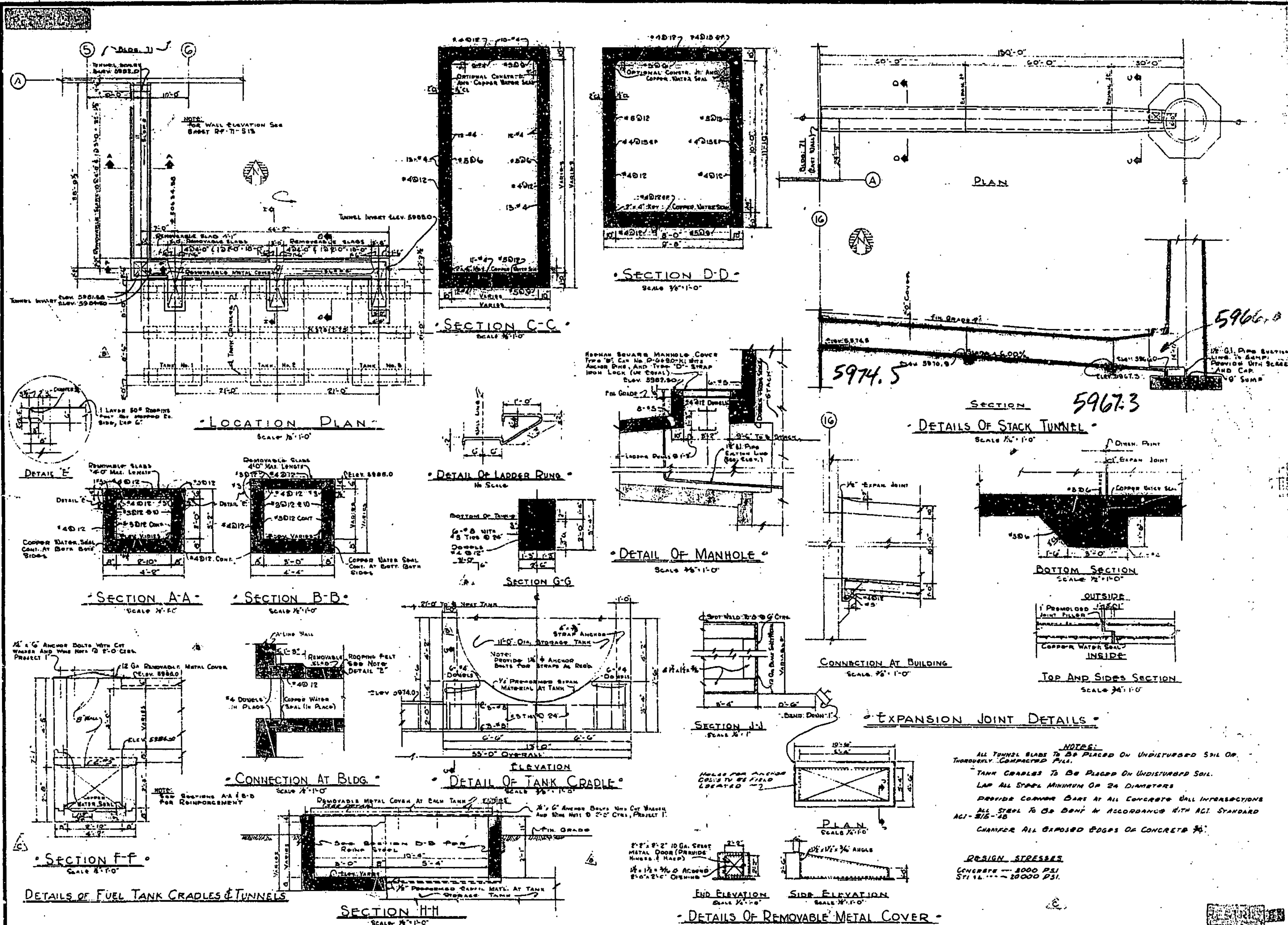


A	AS BUILT - NO CHANGE		
	REVISED DIMENSIONS		
	CHG. LOCATION PLAN & METAL COVERS		
	ISSUED FOR FINAL APPROVAL		
CHANGE ORDER NO.	REVISION	AUSTIN / DOW / A.E.C.	

THE AUSTIN COMPANY
ENGINEERS AND BUILDERS
CLEVELAND
CONTRACT NO. (AT) (29-1) - 1104 W. O. G. - 8001

SUBMITTED J. M. Kinney 6-12-52 THE AUSTIN COMPANY DATE
RECOMMENDED THE DOW CHEMICAL COMPANY DATE
APPROVED 6-13-52 ATOMIC ENERGY COMMISSION DATE

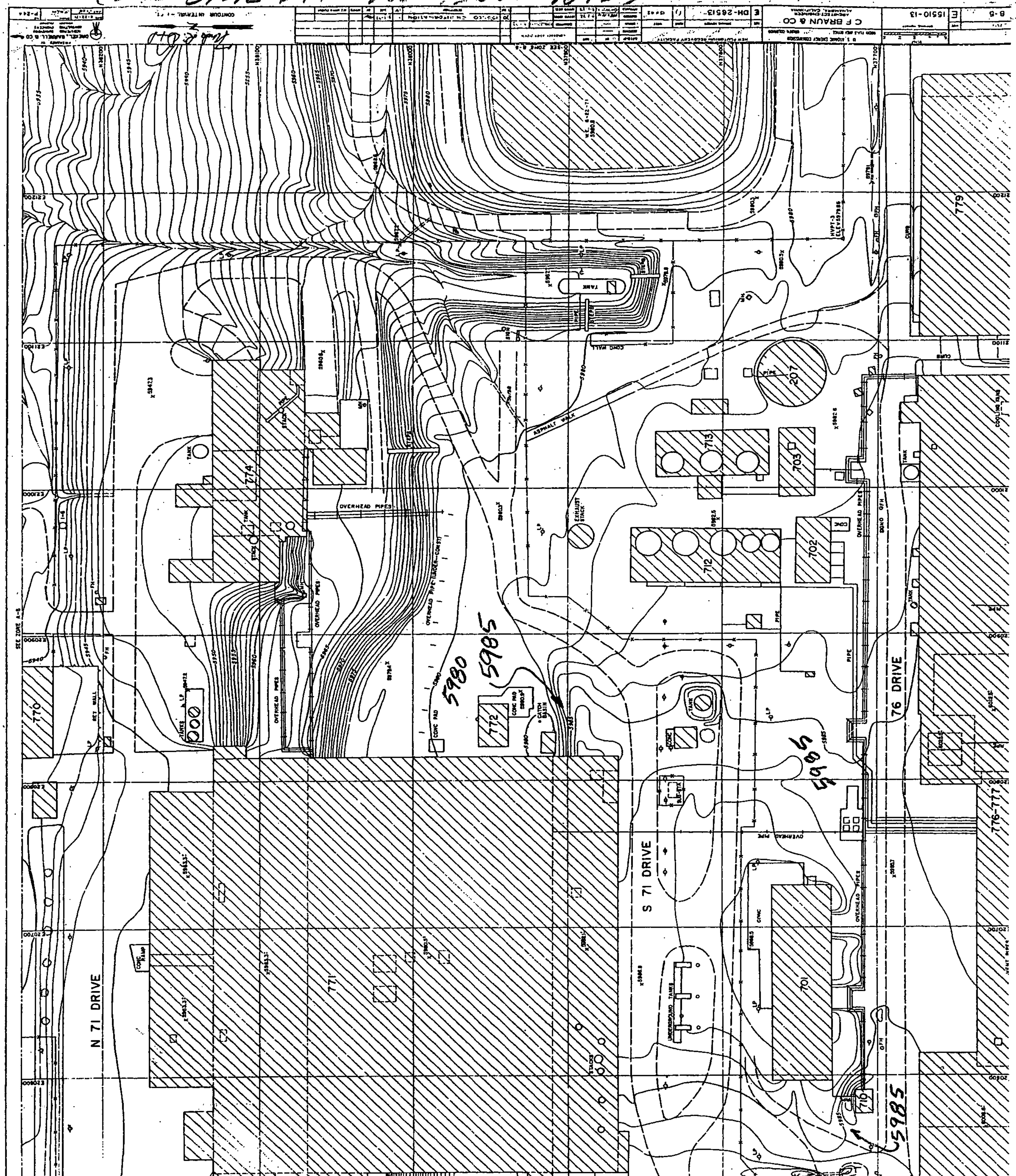
U. S. ATOMIC ENERGY COMMISSION
SANTA FE OPERATIONS OFFICE
LOS ALAMOS NEW MEXICO
ROCKY FLATS PLANT

TUNNEL DETAILS BLDG. 71
DATE 5/11/52 SCALE AS NOTED
DRAWN S: CHECKED
SHEET NO. RF-71-S15-0

(BUILDING DRAWING -00515-001A)

CALC 771-BS-000189 (PAGE 14 OF 21)

(SEE CIVIL DRAWING 15510-1015)



Book + Note

ACI 318-89

ACI 318R-89

Building Code Requirements for Reinforced Concrete (ACI 318-89) and Commentary—ACI 318R-89

aci

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CHAPTER 9 – STRENGTH AND SERVICEABILITY REQUIREMENTS

CODE

COMMENTARY

9.0 – Notation

A_g = gross area of section, sq in.
A'_s = area of compression reinforcement, sq in.
b = width of compression face of member, in.
d = distance from extreme compression fiber to centroid of tension reinforcement, in.
d' = distance from extreme compression fiber to centroid of compression reinforcement, in.
d_s = distance from extreme tension fiber to centroid of tension reinforcement, in.
D = dead loads, or related internal moments and forces
E = load effects of earthquake, or related internal moments and forces
E_c = modulus of elasticity of concrete, psi. See 8.5.1
f'_c = specified compressive strength of concrete, psi
$\sqrt{f'_c}$ = square root of specified compressive strength of concrete, psi
f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi
f_r = modulus of rupture of concrete, psi
f_y = specified yield strength of nonprestressed reinforcement, psi
F = loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces
h = overall thickness of member, in.
H = loads due to weight and pressure of soil, water in soil, or other materials, or related internal moments and forces
I_{cr} = moment of inertia of cracked section transformed to concrete
I_e = effective moment of inertia for computation of deflection
I_g = moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement
ℓ = span length of beam or one-way slab, as defined in 8.7; clear projection of cantilever, in.
ℓ_n = length of clear span in long direction of two-way construction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases
L = live loads, or related internal moments and forces
M_a = maximum moment in member at stage deflection is computed
M_{cr} = cracking moment. See 9.5.2.3
P_b = nominal axial load strength at balanced strain conditions. See 10.3.2

CODE

flection shall be computed with the modulus of elasticity E_c for concrete as specified in 8.5.1 (normal weight or lightweight concrete) and with the effective moment of inertia as follows, but not greater than I_g.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \quad (9\text{-}7)$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \quad (9\text{-}8)$$

and for normal weight concrete,

$$f_r = 7.5\sqrt{f'_c} \quad (9\text{-}9)$$

When lightweight aggregate concrete is used, one of the following modifications shall apply:

(a) When f_{ct} is specified and concrete is proportioned in accordance with 5.2, f_r shall be modified by substituting $f_{ct}/6.7$ for $\sqrt{f'_c}$, but the value of $f_{ct}/6.7$ shall not exceed $\sqrt{f'_c}$.

(b) When f_{ct} is not specified, f_r shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation is permitted if partial sand replacement is used.

9.5.2.4 – For continuous members, effective moment of inertia may be taken as the average of values obtained from Eq. (9-7) for the critical positive and negative moment sections. For prismatic members, effective moment of inertia may be taken as the value obtained from Eq. (9-7) at midspan for simple and continuous spans, and at support for cantilevers.

9.5.2.5 – Unless values are obtained by a more comprehensive analysis, additional long-term deflection resulting from creep and shrinkage of flexural members (normal weight or lightweight concrete) shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor

$$\lambda = \frac{\xi}{1 + 50\rho'} \quad (9\text{-}10)$$

where ρ' shall be the value at midspan for simple and continuous spans, and at support for cantilevers. It is permitted to assume the time-dependent factor ξ for sustained loads to be equal to

COMMENTARY

selected as being sufficiently accurate for use to control deflections.[9.7–9.10] The effective I_e was developed to provide a transition between the upper and lower bounds of I_g and I_{cr} as a function of the ratio M_{cr}/M_a. For most practical cases I_e will be less than I_g.

R9.5.2.4 – For continuous members, the code procedure suggests a simple averaging of I_e values for the positive and negative moment sections. The use of the midspan section properties for continuous prismatic members is considered satisfactory in approximate calculations primarily because the midspan rigidity (including the effect of cracking) has the dominant effect on deflections, as shown by ACI Committee 435[9.10,9.11] and SP-43.[9.4]

R9.5.2.5 – Shrinkage and creep due to sustained loads cause additional "long term deflections" over and above those which occur when loads are first placed on the structure. Such deflections are influenced by temperature, humidity, curing conditions, age at time of loading, quantity of compression reinforcement, magnitude of the sustained load, and other factors. The expression given in this section is considered satisfactory for use with the code procedures for the calculation of immediate deflections, and with the limits given in Table 9.5(b).[9.12] It should also be noted that the deflection computed in accordance with this section is the additional long-term deflection due to the dead load and that portion of the live load which will be sustained for a sufficient period to cause significant time-dependent deflections.

Summary of Integrated Modeling
For the area surrounding Building 771
(Bob Prucha, 9/23/2003)

A localized, high-resolution integrated flow model was developed for the area surrounding Building 771, including Building 774 and 771c. The purpose of the refined model was to simulate local-scale hydraulic conditions due to assumed closure conditions in the Building 771 area. Several hydraulic responses for the closure conditions were evaluated. First, the potential for groundwater levels to rise near, or above the ground surface adjacent to structures left in place was evaluated (i.e., Buildings 771 and 774 basement wall and slab structures, or the tunnels between Building 776 and 771, or 771 and the stack, respectively). The change in hydraulic conditions between current conditions and closure conditions was also evaluated. Finally, potential impacts to the current distribution of Carbon Tetrachloride (IHSS 118.1) were assessed for the assumed closure condition.

The hydraulic response in the above areas to closure assumptions is complex. Saturated zone flows in the vicinity are three-dimensional and complicated by several factors, including the left-in place building structures, an engineered gravel layer beneath building slabs 771 and 774, occurrence of Arapahoe Sandstone, and the hillslope morphology. Given the complexity of the objectives and subsurface flow conditions, and the strong surface-subsurface flow interactions, the integrated code, MIKE SHE, used in the SWWB modeling (KH, 2002), was used to simulate closure conditions and to evaluate the three key hydraulic responses.

The integrated modeling approach was iterative and involved several steps to evaluate key responses. First, an integrated model of the current system configuration (WY2000) was developed. This was done for several reasons. First, it was necessary to obtain a base set of flow conditions that could be used to assess the change in system response due to closure in the Building 771 area. It was also necessary to determine whether model flow parameter values needed adjustment due to the grid scale refinement (i.e., drain leakage coefficients). At the start of modeling, it was uncertain whether drains needed to be included, or eliminated from future simulations so they were included in the model input algorithm.

To a large extent, the same input information used in the SWWB model (KH, 2002) was used to develop the local scale Building 771 model. One exception was the number of saturated zone layers. Seven layers were needed in the refined model, rather than 4 in the SWWB model, to more accurately define the subsurface structures in the area. For example, the tunnel from Building 771 to 776, the explicit definition of the material above the remaining building slab, the building walls and the gravel layer underlying the slab were explicitly defined in the saturated zone portion of the model in addition to the spatially variable unconslidated material and weathered bedrdock. The weathered bedrock definition included both the claystone/siltstone matrix and the sub-cropping and embedded Arapahoe Sandstone distribution. The resolution of the new model is 25 feet by 25 feet, compared to 200 feet by 200 feet in the regional SWWB model.

Only a few simulations of the current configuration were required to obtain a reasonable comparison between simulated and observed hydraulic response. Average annual heads from available wells in the area were compared against simulated average annual heads. In general, simulated heads were well within a meter of observed values. In some locations, heads were greater though, this is likely caused by local conditions not captured in the flow model, even at a 25 foot grid resolution. Average annual discharge estimates from footing drains were also compared against simulated values and found to be within 10 to 20 percent of reported values.

Specific closure configuration modifications were provided by ER personnel. All subsurface pipes (storm, sanitary, footing and water supply) were assumed disrupted, while utility trenches, and hence utility backfill material, was left as is. The ground surface was regraded based on a topographic surface provided by ER. All impervious material at the ground surface was assumed to be removed, while structures less than 3 feet below grade were assumed removed. As a result, the northern portions of Buildings 771 and 774 were removed to accomdate the regrade topography. In addition, Building 771c was assumed entirely removed. Fill material within Buildings 771 and 774 is assumed to be similar to a coarse gravel (i.e., rubblized concrete). Concrete structures left in place below grade were assumed to be leaky (i.e., low permeability).

Results of the closure configuration simulation for a typical climate (WY2000) indicated that average annual water levels are well below 1 meter below grade. Only the area near the northern edge of the remaining Building 771 slab showed water levels nearest the ground surface. As a result, a sensitivity analysis was conducted to further evaluate effects of different assumed parameter values on the hydraulic response in the area. Results of the sensitivity analysis indicated several things.

The most important finding of the sensitivity analysis indicated that a wet year (100 year basis) combined with an assumption that no subsurface drains were operation showed groundwater levels would rise to within a meter of ground surface in a greater area surrounding Building 771. Results also showed, however, that creating holes in the slab floor (decontamination) would only result in an increased head above the slab of about half a meter. Assuming that the Arapahoe Sandstone beneath Building 771 has a hydraulic conductivity similar to the claystone/siltstone matrix, or that the gravel layer underlying the building slabs only resulted in slight increases in head upgradient of Building 771 and 774 walls.

Results of a simulation with conservative closure conditions of a wet year climate coupled with no subsurface drains indicated several things. First, average annual heads upgradient of the Building walls left in place increased, but were still 1 meter below groundsurface. However, for large events during a wet year, results indicate several areas, mostly upgradient and around the southern Building 771 wall, may become saturated at, or near the ground surface. Results also indicated that average annual heads increase above ground surface along the northern edge of the slab for Building 771. As a result, a subsurface collection trench was simulated in the model along the northern edge of the Building 771 and 774 slabs. The trench was simulated to intercept flows both above and below the slabs. Results of this simulation indicated that heads at the slab edge are reduced below ground surface in this area.

The movement of the Carbon Tetrachloride plume (IHSS 118.1) south of Building 771 was simulated using an advection-dispersion module (a DHI code) that uses the integrated MIKE SHE flow model results. It doesn't simulate effects of degradation, diffusion, or sorption, but it accounts for most of the plume movement from the assumed source area. Assuming the slab-drain, along the northern slab edge of Building 771 and 774 extends to the top of the weathered bedrock transport simulations showed that plume moves to the west and north (and below Building 771). Neither the western, nor the northern plume extents are fully captured with this assumed drain depth, but they do appear to be captured if the drain depth is assumed constant at 10 meters. A western arm was assumed based on discussions with ER that extends from the western edge of Building 771 down south.

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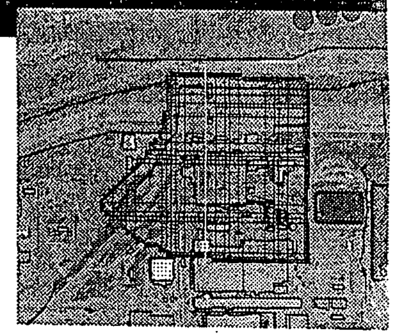
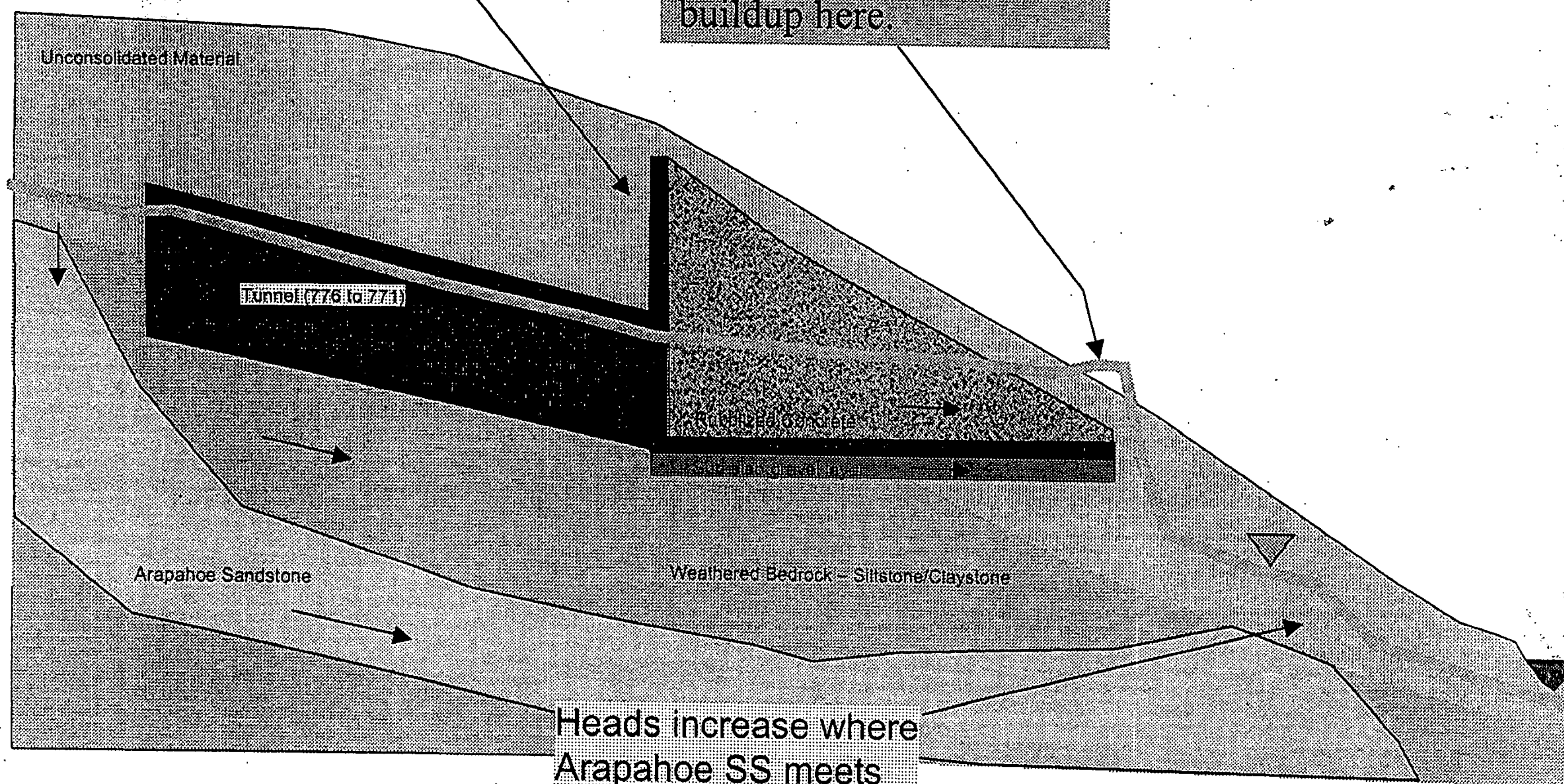
Integrated Modeling – Closure Configuration (WY2000) Basecase (Typical Climate) - No slab drain



Average annual heads behind wall > 1 m deep

NO DRAIN. Heads can buildup here.



Heads increase where Arapahoe SS meets Unconsolidated material

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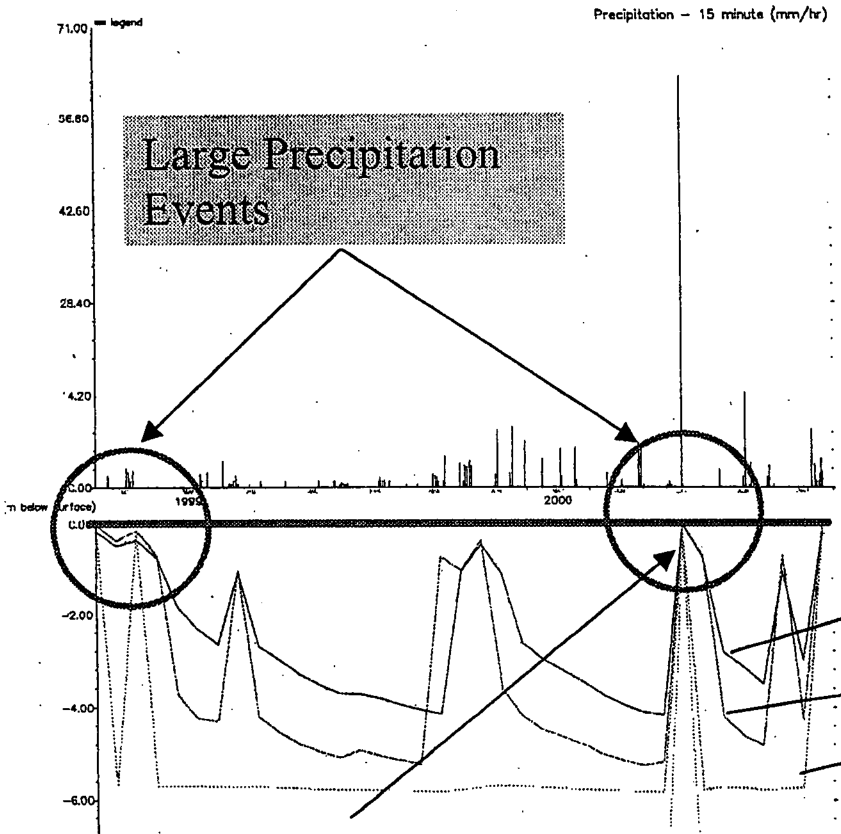
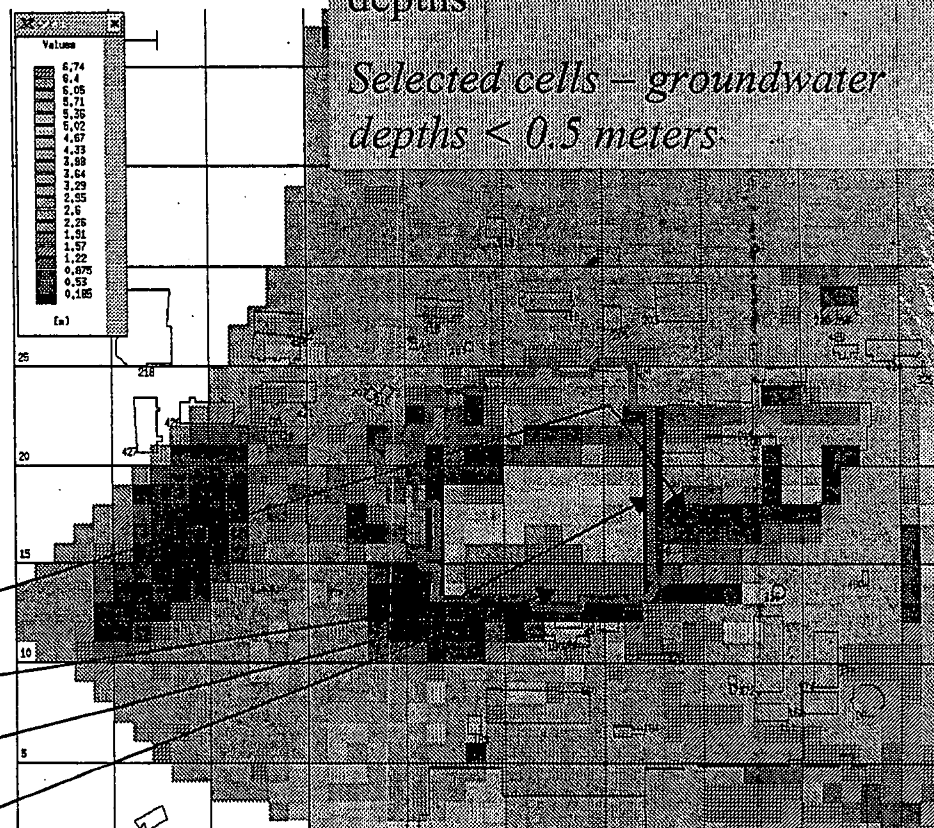
Integrated Modeling – Closure Configuration
Groundwater Depths – Seasonal Effects



Large Precipitation Events

Groundwater depths may reach ground surface - large events during wet year



Minimum annual groundwater depths

Selected cells – groundwater depths < 0.5 meters

1) These are infrequent events.

2) These areas may require maintenance.